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SOME PRACTICAL ASPECTS OF SAND DRAIN STABILIZATION

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SOIL MECHANICS AND FOUNDATIONS DIVISION

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SOME PRACTICAL ASPECTS OF SAND DRAIN STABILIZATION

Stephen M. Olko,[1] J.M. ASCE

SYNOPSIS

Sand drain stabilization of soils has increased in recent years as a result of the necessity of constructing highway and airfield embankments on unstable, compressible soils. This paper presents, in a general manner, some practical aspects of sand drain design, installation and field control.

INTRODUCTION

The Soils Engineer is often confronted with problems of embankment stabilization. Although there are a number of stabilization methods employed, such as excavation, displacement, surcharging, electro-osmosis, etc., each has its limitations—depending on economics, soil characteristics and time available for construction.

Stabilization, by means of sand drains, has proven itself to be economical and satisfactory for certain conditions. Sand drains have also been used in the past to relieve excess hydrostatic pressures and to increase the shear strengths of soils in work not necessarily requiring stabilization of highway or airfield embankments.

The purpose of this paper is to present some of the field problems attendant with a sand drain installation. A better evaluation can then be made when considering this method as a solution to a particular problem.

The usual procedure employed in installing a sand drain and the manner in which it functions is as follows: First, a layer of pervious sand, called a "blanket", is placed on the swamp surface as shown on Figure 1. The sand blanket supports construction equipment and later provides horizontal drainage for the escape of pore water. A hollow mandrel is then driven into the soft soil, filled with coarse sand, air pressure is applied and the mandrel is extracted—leaving behind a vertical sand drain. Construction of the embankment then proceeds in controlled increments of loading until a height above finished grade elevation is attained, including a predetermined overload as shown on Figure 2. The overload insures full settlement of the embankment within the construction time available and is removed upon completion of stabilization. The excess overload material may be employed to trim slopes, or as fill at other sections of the embankment.

The rate of construction is controlled by observations on settlement plates, side stakes and pore water measuring devices shown in Figures 1 and 3.

Initially, the weight of a load increment is carried by the pore water contained within the relatively impervious claylike foundation soil. The difference in hydrostatic pressure between the pore water in the pervious sand drain and the clay-soil, causes a flow towards the sand drain. Consequently, the excess

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hydrostatic pressure in the clay-soil is reduced and, as a result of decreasing water content, consolidation occurs and the shear strength of the soil is increased. The rate of consolidation with sand drains is greatly increased since it is a function of the distance a particle of water must travel to an area of neutral pressure. With sand drains a water particle has to travel but a few feet horizontally to a pervious sand drain, rather than travel distances many times longer, such as vertically upward to the ground surface. Obviously, the closer the spacing and larger the diameter of drain, the greater will be the theoretical rate of consolidation.

Borings and Testing

Soil borings and laboratory tests are essential for the design of a sand drain installation. Borings with continuous undisturbed samples are needed to obtain representative samples for laboratory testing. Other borings, with driven split tube samples, are required for the determination of the soil stratification from which may be estimated sand drain lengths and drainage conditions. Probings may be employed to estimated sand drain lengths in areas of known soil stratification.

The laboratory testing program is rather involved. Correlation indices, such as water content and Atterberg limits, permit determination of variations in soil characteristics and reduce the number of other expensive and time-consuming tests required. The prime tests, however, are consolidation, unconfined compression, consolidated - quick triaxial, vertical and horizontal permeability, and remolding tests for sensitivity. From these tests the sand drain spacing and diameter are established and the settlement and stability are estimated for a given height of embankment. The permissible rate of construction and overload requirements are also estimated from the test results but are later verified in the field by observation. The laboratory tests and analysis serve only as a guide. The behavior of the soil in the field determines the actual rate of construction.

Design

In the following discussion, which concerns itself with the design of sand drains, the details of stability and settlement analysis are avoided. Although such items as secondary time effects, validity of slide analysis, etc., are important, they will not be discussed in this paper as these problems are not peculiar to sand drain installations alone. Instead, the general theory of design and some practical aspects of time scheduling will be discussed.

General.— The theory of sand drain design is presented in detail in the various standard texts and the 1948 Transactions of The A.S.C.E. Briefly, the analysis consists of determining separately the consolidating effect of horizontal flow of pore water to a sand drain and the simultaneous consolidating effect of vertical flow of pore water to both the ground surface and underlying pervious strata. From the "% Consolidation" values determined for each of these two separate flow directions, an "Overall % Consolidation" value is obtained. Settlement and increase in shear strength are then determined in the conventional manner, from the "Overall % Consolidation" value —as if only vertical drainage occurred. Graphs can be plotted, as shown on Figure 2, superimposing the effects of increment loading and showing settlement vs. time.

The graphical plots will have to be based on some initial assumptions, as for example, an allowable loading rate of one foot every two weeks and not less than 95% consolidation of the completed roadway at finished grade, upon

removal of overload. These factors must be correlated with embankment stability, permissible settlements of the paved roadway, and conformance with the desired specification requirements for loading. In addition, loss of effective weight as consolidation proceeds must be considered since portions of the embankment will become submerged below the water level of the swamp. Also, since settlement occurs between applications of load increments, a decision must be made as to whether the contractor will be permitted by the specifications to place increments of fill in elevation, or in increments of height. An appreciable difference in construction time and loading will result, depending on the wording used in the specifications.

Time Schedulings.— Considering the possible variations in rate of loading, overload, sand drain diameter and spacing, it is apparent that many variables exist and that the time required for a given amount of settlement to occur can range between wide limits.

Several of the toll roads constructed in recent years have sections stabilized with sand drains. These roads had to be completed and opened to traffic on a specified date in order to meet requirements of bond issue, legislation, and payment of interest. An estimate of the date at which the contract will be signed and the date at which overload must be removed for paving, yields the total time available for sand drain driving, placing of load increments, and settling of the fill with overload. The design required to accomplish this is established from a study of soil characteristics, economy and availability of soil borrow.

Economical and rapid construction is achieved when a soil is relatively pervious and shear strength increases rapidly with consolidation. The Contractor's forces are then employed full time and efficient operations result. However, where the soil is relatively impervious and the rate of shear strength gain is small, construction is very slow as it may be necessary to delay placing additional load increments for several weeks to insure stability. As a result of this delay, the total time required for sand drain stabilization will increase. The costs will also correspondingly increase because of contract extension, inefficient use of manpower and loss of revenue from the completed highway or airfield.

Where the soil does not gain shear strength as rapidly as predicted from the laboratory tests, it may still be possible, with slow stage construction, to attain overload elevation within the total construction time available. This would be accomplished at the expense of reducing the time of consolidation with overload and, upon removal of the overload, the full degree of consolidation required under the finished subgrade elevation will not have been attained. Consequently, the paved roadway will continue to settle excessively in the future with high maintenance costs and disruption of traffic.

It is possible sometimes to obtain the necessary degree of consolidation by continuing construction of the fill to a higher overload elevation than originally assumed and utilizing the increased weight to obtain additional settlement, prior to removal of overload. This procedure is not entirely satisfactory as the higher embankment may prove more unstable. In addition, the weight and cost of the extra overload is not fully exploited as it has to be removed soon after being placed. As the embankment elevation increases, the consolidating benefits of each successive increment are reduced because of the shorter period of time remaining before the overload is removed and paving commences. Some saving is realized in that lesser quantities of material are placed in the upper increments of the embankment prism, as compared to the quantities placed at lower elevations. These savings however, are overbalanced by the necessity of removing later a greater quantity of overload and increasing the problems of overload material disposal.

The overload may be removed when sufficient consolidation has occurred. This is evidenced by reductions in both the pore water pressures and the rate of settlement. The values must be plotted to indicate rates of decrease and stabilizing trends.

A theoretical method of hastening consolidation and stabilization within the construction time available is to increase the diameter of sand drains and reduce the spacing. There are, however, both economical and physical limitations to this procedure. From an economical viewpoint, the costs become prohibitive in many areas where coarse sands are not readily available and it is necessary to use expensive washed sands. Employing large quantities of such materials increases the cost of sand drain stabilization. It may then be more economical to remove the unstable soil by hydraulic dredges or other means and replace it with less expensive soil, but of poorer quality than would be used for sand drain material. Also, the cost of driving additional drains must be considered. In some instances a complete realignment condemning more expensive property may prove economical.

From the physical viewpoint there is a practical limit to the diameter and spacing of sand drains. Using large diameter sand drains, spaced closely together, would tend to displace and remold the soil to such an extent as to excessively reduce the original shear strength. The horizontal permeability would also be decreased since the remolding tends to destroy the continuity of any thin silt and sand drainage layers. Severe displacement of the soil would tend to shear adjacent sand drains and prevent continuous vertical drainage. It is apparent, therefore, that the effects of increasing the sand drain diameter and decreasing the spacing may not hasten consolidation once a certain physical limit is reached. Based on past experience, the practical limit appears to be sand drains of 20 inches diameter, spaced about 6 feet apart. However, the limiting spacing and diameter is a function of the soil's sensitivity to remolding. Although theory indicates that large diameter drains spaced closely together increase consolidation and stability, the opposite may be true. Smaller diameter drains spaced further apart are more economical and may function more efficiently in sensitive soils. Research is greatly needed to resolve this question.

The use of counterberms, or the flattening of side slopes, are other methods of obtaining required settlement within the time available for construction—when the gain in shear strength, at a fast rate of construction, is insufficient to insure a stable embankment. Here again, however, there are both economical and practical limitations. Besides the cost of the counterberm material itself, it will be necessary to consider the added cost of extending drainage beneath the counterberm for drainage continuity. An alternate scheme is to use some method of drainage to supplement the sand blanket, such as porous concrete pipe inside a gravel drain, as shown on Figure 4. If the counterberm is very wide the cost will be further increased by the purchase of additional land for right-of-way. In some cases, sand drains may be required beneath the berms for stability, as the practical height of counterberm construction is limited by the stability of the berm itself. Placing sand drains beneath a berm increases costs but has the advantage of increasing the shear strength of the soil in a region of probable slide failure, at the embankment toe. The berm weight provides the necessary consolidating load and counterweight.

Disadvantages attendant with berm construction are that they complicate the drainage system and later become a problem of disposal and timing. This is particularly true if the berm material is to be used later at another roadway section, after the natural soil has gained sufficient shear strength to support the embankment itself. It is therefore apparent that while construction of flatter side slopes or berms permits faster construction, it

also increases the costs and complicates the design.

The use of wellpoints inside sand drains is still another method that can be employed to increase the shear strength of a soil and thus permit more rapid construction of an embankment. Although wellpoints have never been used extensively on any roadway section because of the equipment and pumping costs, they have been employed at critical locations to arrest potential failures. The common method of installation is to simply jet the wellpoint into the sand drain with a small pressure. Supporting the wellpoint vertically during installation and observing the color of the wash water gives some assurance that the wellpoint will be located within the drain.

The main purpose of using wellpoints is to lower the water head inside the sand drain and thus hasten consolidation by increasing the flow of pore water. In an ideal situation, where a pervious sand layer lies beneath the unstable soil at a reasonable pumping depth, the wellpoints may be extended into the pervious layer. The sand drains will then drain water vertically downward and it may be unnecessary to place a wellpoint in each individual drain. An added benefit of this scheme is that the effective weight of the soil may be increased, which in effect would superimpose an additional load to aid consolidation. The principle of increasing the effective weight can be further extended by creating a vacuum in the sand drain and utilizing atmospheric pressure as a load. To accomplish this it would be necessary to seal off the drain below the bottom of the sand blanket.

In the design of a wellpoint installation, consideration must be given to the potentially unstable conditions caused by seepage pressures and sudden drawndown effects. It may therefore be necessary at the beginning to pump slowly in stages in order not to excessively increase the effective loads. Observation wells may be required to determine ground water elevations and in some instances pumping may have to be restricted to the sand blanket alone. The purpose of this restriction is to decrease the build-up of water table in the sand blanket itself, due to the upward flow of water from the sand drains, and yet eliminate the possibility of too large an increase in effective soil pressures. The wellpoints should be located ideally where the seepage pressures, reduction in water head, and increase in effective soil pressures all tend to increase both the stability and the rate of consolidation. As a practical matter this may be impossible and a compromise solution is necessary.

Wellpoints have been successfully used in the past with sand drains at critical locations. Their use has been limited because of cost, maintenance, interference with other operations and curtailment of embankment construction in the region of installation. It must be realized that it is not uncommon to pump several months before stabilization is accomplished.

Wellpoints are not an economical substitute for providing load, when compared to placing of earth fill, since the embankment must be constructed up to design elevation anyway. Usually the most economical means of providing the necessary consolidating load is the weight of the embankment itself.

Installation of Sand Drains

To illustrate the various phases of field control and associated problems, a typical sand drain installation will now be described in detail, assuming the construction of an embankment in a swamp area.

Clearing and Drainage.— The first work under contract would be clearing, grubbing and drainage. The area should be cleared of tree stumps and tall grasses. Great care should be exercised not to destroy the existing meadow mat. The meadow mat, consisting of a fibrous flexible layer of roots, provides support for the sand blanket and sand drain driving rigs during the initial stages of construction. Destruction of this mat would increase

construction problems and severely limit the thickness of sand blanket which could be placed prior to driving of drains.

Surface drainage is necessary to divert all water away from the area of embankment construction. This prevents increase of ground water elevation in the sand blanket and consequent reduction in the rate of sand drain consolidation.

Construction of a major culvert to provide flow through a sand drain stabilized embankment presents a number of interesting problems. A permanent rigid culvert cannot be placed directly on the swamp surface as it would crack excessively from the differential settlements occurring across the embankment cross section. Constructing the culvert on piles driven to firm bearing may be rather expensive as the pile driver probably could not reach the site until after the sand blanket had been placed. Costs would increase as a result of excavating the sand blanket, pumping, and employing material and equipment which may not be common to the remainder of the work. In addition, the piles would have to support not only the weight of culvert and overload, but also the weight of the compressible soil in which they are driven. The soil would have a tendency to transfer its weight to the piles by skin friction, due to consolidation under the weight of the embankment placed on either side of the culvert. The maximum negative friction force developed could be equal to the shear strength of the soil itself. A lower bearing capacity would have to be assigned to each pile and therefore a larger number of piles would be required to support the total culvert load.

A method of constructing large culverts is to first divert the stream by means of temporary flexible culverts. Meanwhile, sand drain construction can proceed at the permanent culvert site, with decreased sand drain spacing and perhaps even increased embankment overload. The purpose of these design changes is to hasten consolidation so that full settlement may be achieved prior to the remainder of work under contract. Sufficient time is then available to excavate the embankment and construct the permanent culvert, without increasing the total time of contract. After consolidation has been achieved at the culvert site, the flow is diverted into the permanent concrete culvert. The temporary flexible culverts, which have been distorted by the differential settlements, are then removed. An alternate scheme is to fill the temporary culverts with sand, blown in under pressure, and seal the ends with concrete.

In some instances, where the depth of compressible soil is not great, it may be more economical to excavate the soft soil and replace it with granular backfill. Culvert construction can then proceed on the backfill. The excavation and backfill quantities may be decreased by using a sheeted excavation and salvaging the sheet piling upon completion of work.

It is apparent from the above that the construction of culverts through a sand drain stabilized embankment must take into account availability of materials and equipment, soil stratification and characteristics, and also, time scheduling.

Sand Blanket.— The second item of work is the placing of the sand blanket which supports the sand drain driving rigs and provides horizontal escape of pore water from the drains. The contractor may be permitted to place on top of the sand blanket one or two feet of embankment material as a "working table" for the equipment.

The thickness of sand blanket varies from approximately 2 to 7 feet and it often extends at least 5 feet beyond the toe of embankment. The required thickness of sand blanket is a function of drainage, soil strength, and availability and cost of material. Obviously, too great a thickness cannot be placed prior to the driving of sand drains as a slide failure of the blanket itself would result. The thickness is also limited by the economic availability of suitable

coarse sand. A thick blanket may also increase the total construction cost since longer sand drains are required and therefore, the costs of driving will be correspondingly increased. This is particularly true where bids are unbalanced and embankment is to be stabilized over relatively shallow swamp areas.

Sand blanket material is often specified by gradation, rather than permeability. The usual limiting value is 3% passing the number 200 sieve. The material is considered too impervious if a larger percentage were permitted. Research is needed to determine optimum thickness, gradation and permeability for given permeabilities of sand drains.

Sand blanket material may be placed "dry" by truck and bulldozers or placed hydraulically. When it is placed by truck, the problem of field control is relatively simple since borrow pits can be inspected and explored to insure conformance with specifications. Hydraulic placing, however, imposes several additional problems of field control. In order to retain the hydraulicly placed sand blanket it is necessary to construct dikes. Building such dikes from swamp material excavated at the toe of the sand blanket should be discouraged. The relatively impervious swamp soil is a permanent barrier to the escape of pore water through the sand blanket, unless it is later pierced by drainage openings, as shown on Fibure 4, or completely removed from the job site. In addition, excavating the swamp soil at the toe of sand blanket destroys the meadow mat and tends to induce mud waves and slides. The dikes, therefore, should be constructed preferably of pervious material or removable flash boards.

Because of the nature of hydraulic filling, it is extremely difficult to control the gradation of material as areas are often inaccessable during pumping. It may be found that localized pockets of the sand blanket contain too many fines. In such cases, the material may have to be removed and replaced, or mixed in place with coarser soils. Another alternate is to place gravel drains in the sand blanket, particularly from the center of the roadway, where the water has to travel a relatively long distance to escape. Gravel drains may also be required where the sand blanket extends into a natural valley and is surrounded by high ground on three sides. Drainage through the sand blanket alone would be extremely slow in such an area. In some instances it may be desirable to place drain pipes within the gravel drains to increase the flow.

Sand Drains.—The third item of work is the driving of sand drains. Drains usually vary from 12 to 20 inches in diameter, with spacings of 6 to 20 feet and are driven to depths of 10 to 100 feet.

The spacing of sand drains is usually determined from settlement and stability analysis based on laboratory soil tests. The sand drains must extend a sufficient distance beneath the toe of embankment to stabilize this critical region and to intercept excess pore pressures transmitted horizontally from beneath the center of the embankment. The excess pressures are more pronounced in stratified soils and reduce the natural shear strength of the soil at the embankment toe by decreasing the effective soil stresses.

The linear footage of sand drains driven per shift varies considerably and is dependent upon spacing, soil resistance and thickness of swamp deposit. In addition, the mobility of the equipment and the placing of mats for support have to be considered. Assuming a spacing of 7 feet and a depth of 45 feet, driving an average of 2,500 linear feet in a shift with one rig is considered good progress, although there have been reports of over 6,000 linear feet driven in a shift.

The cost of sand drains varies widely, depending on the availability of material, the spacing, depth and driving resistance. Prices ranging from $0.40 to $1.50 or more per foot have been reported.

Sand drains have been installed in the past by means of rotary drilling and/or jetting. However, they are usually installed by means of a driven mandrel, as shown on Figure 5. This is a relatively fast method although difficulties are sometimes experienced in driving through sand layers and the driving vibrations may be harmful to structures in the vicinity. On the other hand, jetting requires large quantities of wash water, tends to build up a water table in the sand blanket and is difficult to control.

The bottom of the mandrel is closed during driving with a hinged plate, a plate on a chain, or concrete or wood plugs which are left in the ground. The purpose of any of these devices is to prevent entrance of soil during driving and permit forcing of the sand out of the mandrel during extraction.

There have been reasons advanced to drive the mandrel open-end, blow out the soil, and then fill the mandrel with sand. This method has been proposed and used on some projects, partly on the basis that the soil is less disturbed and that horizontal permeability is increased, since the continuity of sand or silt layers is retained. With increased horizontal permeability less sand drains of smaller diameter may be required and this may, in part, pay for the additional operation of blowing the mandrel clean—or if the same spacing and diameter are retained, the added costs would be charged to completing stabilization at an earlier date. Depending on the soil stratification and sensitivity, it is possible that open-end mandrels may be a solution to some problems of time schedule, if the costs could be reduced. However, it can easily be visualized that working conditions may become unpleasant when a half dozen rigs are simultaneously blowing out muck through 90-foot mandrels. Disposal of the muck and silting of the sand blanket must be considered.

The sand drain material is, as a rule, coarser than the blanket material. Usually no more than 3% is permitted to pass the number 100 sieve. The sand drain material must be sufficiently pervious to offer little flow resistance and also act as a filter material to prevent migration of "fines", particularly in relatively non-cohesive silt soils. To obtain such material it may be necessary to employ washed sands. In some instances substitute materials, such as slag, may prove economical and function as well as natural sand. However, the properties of the substitute material should be carefully investigated before acceptance.

The sand is placed in the mandrel by means of a "skip", as shown on Figure 6. The skip hoists the material to the top of the mandrel, it is then tipped and the sand is dumped through a Y connection and past a flap valve, often constructed of rubber, backed with thin plates. Air pressure is then applied on the sand to assist extraction of the mandrel and to prevent the sand from arching and coming up with the mandrel. Oftentimes the sand is placed during the last stages of driving as a means of speeding progress and to insure that the sand fills the entire mandrel without arching at some elevation above the bottom. The vibrations from driving prevent arching. Checks should be made on the volume of sand placed in the skip hoist and the volume of material required to fill the sand drain. There should always be a slight excess of material upon extraction of the mandrel.

The pressure required to keep the sand in place can best be determined by trial. In some cases pressures of 100 pounds per square inch are necessary. The pressure should not be too high because of the dangers associated with material blown about upon removal of the mandrel. With high pressures the meadow mat may be lifted and the remaining sand blown beneath the mat as the bottom of the mandrel approaches the surface. It is therefore desirable to reduce the pressure near the surface. Some installations accomplish this by providing a hole in the mandrel, a few feet above the bottom, so as to partially release pressure when the hole is first exposed above the ground surface during extraction.

A few contractors in the past have expressed a preference to use steam, rather than air, to keep the sand in place. Steam, however, cannot be used successfully unless the contractor has available large boiler capacity. The steam rapidly condenses when it enters the relatively large diameter mandrel and extraction becomes a slow procedure, waiting for the steam pressure to build up.

The depth to which the sand drains should be driven is a problem not only from a payment viewpoint, since sand drains are usually paid on a footage basis, but is also a soil mechanics problem. Depth marks are often welded on the mandrel, which simplifies the record keeping, but the required penetration is determined from the borings which show the soil stratification and compactness. It is, of course, desirable to drive the mandrel through the compressible soils. Where pervious soils exist beneath, the mandrel may be driven deeper to obtain the added benefits of double drainage. For purposes of field control an elevation or blow-count criteria must be established.

The blow-count criteria is established by comparisons with the boring data and should be reviewed frequently as the work progresses to new areas. Overdriving is undesirable as the cost of the sand drain increases, when payment is based on footage, and the time of driving is also increased. In addition, overdriving tends to damage the equipment which causes delays for repairs and adds to costs.

In connection with sand drain stabilization, it should be mentioned that Walter Kjellman of the Royal Swedish Geotechnical Institute has developed a machine which operates in a manner similar to a sewing machine, driving cardboard wicks into the soil. These wicks are spaced relatively close together and function as sand drains.

Embankment Construction.— The fourth item of work is the placing of the embankment in layers, the rate of construction depending on the rate of gain in shear strength. This can be accomplished hydraulically or by the usual trucks and pans. Hydraulic operations are more economical for the wider, lower increments. No compaction of fill is specified with hydraulic placing, where the usual limiting specification is 5% passing the number 200 sieve. When the fill is placed by truck, however, it is specified to be compacted—usually 90% modified AASHO minimum.

A well compacted fill has all of the advantages associated with such a material, namely, high shear strength and good load distribution characteristics. The disadvantages are increased costs and the fact that due to the large magnitudes of settlement and corresponding differential settlement associated with sand drain stabilization, cracks may form in the surface and the deeper portions of embankment are bound to undergo expansion and loss of density. Such cracks may permit entrance of rain water and are potential slide surfaces.

Attaining very high densities may therefore be undesirable. Certainly the consolidation of the embankment material itself is a small consideration since the soil is preconsolidated by the overload. The magnitude of settlement occurring in the fill material is relatively small in comparison to the settlement of the underlying compressible soil. A logical approach to the problem would be perhaps to permit less compaction of the embankment section permanently located above the water table. The upper lifts may be compacted in accordance with present standards; whereas, the upper portion of the overload, which is later removed, need not be compacted any more than is necessary to permit movement of construction equipment.

Where stability of the upper embankment sections is critical, it may be desirable to employ lightweight aggregates. The unbalancing forces are then reduced and the possibility of failure is decreased.

The total quantities of fill material required are the summation of the volumes of settlement, embankment cross section from the original ground elevation, and overload. In constructing the embankment it is necessary to start with a width greater than the design width required at original ground elevation. The purpose is to compensate for settlement and to take into account the fact that the width of the embankment trapezoid, at a given elevation, decreases with the settlement. The design slopes are achieved by starting initially with a greater width. Increasing the embankment widths during construction to compensate for settlement is undesirable. Additional material would be added to the side slopes and this would cover the exposed portions of the sand blanket, hindering free drainage—or the slope would have to be steepened and stability decreased.

Field Control

The following discussion deals with the methods employed to determine when an additional load increment may be added.

Visual observations, settlement plates, side stakes and pore pressure devices are all used together in arriving at a decision as to whether or not the next load increment may be added. The laboratory test results and theoretical analysis serve only as a guide to the probable safe rate of construction.

Visual Observations.— The first item, visual observations, cannot be overemphasized. There is absolutely no substitute for "walking the line" each day, observing the embankment condition, the pore pressures and determining the reasons for any apparent absence of consolidation.

Settlement Plates.— The second item, settlement plates, are used to establish the rate of consolidation and sometimes payment quantities. A decreasing settlement rate indicates that stabilization of the embankment is occurring.

Settlement plates, about 3 feet square, are constructed of timber or 3/8-inch metal plate. A merked vertical pipe is extended upward as the fill height progresses, permitting measurements of settlement and fill quantities. The plates are often spaced about 200 feet apart, with 3 plates across the roadway to determine the settlement cross section. Elevation readings are taken about once a week. Settlement plates are either set directly on the swamp surface, prior to placing of the sand blanket, or are set on top of the sand blanket. The best location is established from a consideration of the contract provisions for payment of fill and the specified control for placing of sand blanket. It has been observed that plates set directly on the swamp surface sometimes have a tendency to indicate less material than was actually placed.

Side Stakes.— The third item, side stakes, are also used to estimate the feasibility of adding another load increment. Side stakes measure horizontal and vertical movements beyond the embankment toe of slope. These stakes detect the formation of mud waves and impending slide failure. They are usually spaced about 100 feet apart, parallel to the embankment. The distance from the toe of slope is a matter of judgment depending on the height of embankment, depth of compressible soil, and an estimate as to where a mud wave might develop. The distance may be 40 to 50 feet. In some installations 2 rows of side stakes are placed. The theory is that a mud wave may start at a greater distance from the toe of slope and not be detected by the first stake or that the center of rotation of a slide failure may occur at the first stake, with very little evidence of movement. Where two rows of stakes

are used they may be spaced, say, 30 and 80 feet from the sand blanket toe of slope. At critical sections it may be desirable to place side stakes in the embankment slope as a further check on relative horizontal movement.

Side stakes are often constructed of timber and consist of a vertical 2x4, and a horizontal cross piece, driven several feet into the swamp surface. Vertical movements are determined from elevation readings. Horizontal movements are determined by aligning a transit between two fixed points and sighting on the graduated cross-piece. In some installations, only the vertical timbers are used, initially aligned in place by transit. Relative horizontal movements are then detected visually by sighting along the stakes. However, the magnitude of movement is unknown and records cannot be kept.

Installing side stakes and taking readings involves a considerable amount of surveying as reference points must be established outside the influence of embankment settlement. Several sight lines are required along curved portions of roadway. The intervals at which side stake readings should be taken are determined by the critical stages of construction. It may be found necessary to take readings each day. At least one reading should be taken for each increment of load application.

Pore Water Measuring Devices.— Pore water pressure measuring devices are the fourth means of determining permissable rates of construction. They indicate the degree of consolidation occurring from measurements of the hydrostatic pressures of sub-soil pore water. The spacing of these devices is often about 200 feet along the roadway center with an additional one placed alternately at each shoulder. Often all penetrate to a constant depth with some check installations at various other depths for correlation purposes. They can be installed prior to, or after, placing of the sand blanket.

Pore water pressure devices have been assigned in the past with alternate uses and combinations of materials. Neglecting electric-diaphragm cells, there are 3 basic types of pressure devices—namely, the vertical stand pipe, the single line with gage, and the double line with gauge, as shown on Figure 3. The installation of each should be carefully supervised to insure tight connections and elimination of air in the tubings. When constructed of metal pipe, the device may be driven or jacked into the soil. Oftentimes, however, a casing is first driven and then cleaned. Next, the device is installed, backfilled with pervious sand, to prevent clogging of the point, and sealed with bentonite. Samples of the soil may be taken to verify soil conditions as the casing is installed.

The vertical stand pipe is usually nothing more than a pipe with a wellpoint at the bottom installed in a sand filter and sealed with bentonite. Readings of pore pressures are obtained by observing the elevation of water in the pipe. The advantages of this method are that elevations may be measured directly and there is little chance of mechanical failure. Such installations, however, are not sensitive to slight changes of pressure and interfere with construction operations since they project above the fill surface.

A more sensitive pore water measuring device, developed by Prof. A. Casagrande, and used at Logan International Airport was constructed of nonmetallic plastic saran tubing and Norton porous tube. The elevation of the water in the tube was measured directly by an electric sounding device.

The second type of measuring device, the single line with gage, is widely used. In simple form it may consist of only a 1/2-inch vertical galvanized pipe with wellpoint and a gage on top. In some installations flexible 3/8-inch copper tubing or plastic saran tubing have been employed. The tubes are laid on the ground surface with gage connections off to the roadway side. This arrangement eliminates interference with construction operations. The lines should be laid with some horizontal loops to provide extra length of tubing as the embankment settles and distorts.

Two important details of design with the single line system are the provisions of a relief valve for bleeding of gases collecting in the tubing and the tilting of the gage stem downward to prevent gases from collecting inside the gage. The gage itself should be a high quality, outdoor type, and should record both positive and negative pressures. The range is usually 30 psi but is dependent on the height of embankment and pressures anticipated.

The third type of pore pressure measuring device is the double line with gage. It consists essentially of a galvanized pipe with a wellpoint, connected at the top with two flexible lines of copper tubing, one above the other, leading out horizontally off the roadway. The lower line is connected to a gage and has a relief valve similar to the single line system previously described. The upper line contains only a relief valve at the end, as shown on Figure 7. The advantage of the double line system is that the entire installation may be cleared of gases by pumping water or kerosene into the lower line and permitting the gases to escape through the upper line. The small diameter tubing makes this possible in that the gas bubbles are pushed ahead of the pumped liquid. The double line also serves as a means of determining fractured tubing.

It is necessary to correct the gage pressure when an estimate of absolute pressure is desired. The correction must take into account the height of gage above ground water, tidal and barometric pressure variations, the weight of fill recently placed on the ground surface, which may still be supported by pressure in the water, and when installed in winter, the specific gravity and quantity of kerosene pumped into the system.

The gage readings are a guide to the consolidation occurring and the resulting gain in shear strength. Decreasing pore pressures indicate that portions of the load are being transferred to the soil structure and assist in determining the possibilities of placing additional load without inducing failure. When the embankment is constructed in a series of stages, it may be desirable to delay placing of the next load increment until the pore pressures have decreased considerably from the total increase resulting from placing of the previous load increment. On the other hand, if the embankment is constructed at a uniform rate, a criteria can be established whereby a specified pore pressure is not to be exceeded, depending on the laboratory tests and the design analysis. The test results, however, should serve only as a preliminary guide since the actual behavior of the soil in the field controls and establishes the construction rate.

It will be found that gage pressures may vary erratically, individually, or when compared with each other. This is caused by gas pressures, variable heights of fill, pile driving operations, construction equipment loads, changes and natural variations in soil permeability and the proximity of sand drains. It is therefore necessary to employ judgment in the interpretation of the readings and to consider all field conditions together.

Readings of pore pressure should be taken quite frequently, perhaps a few times a week, with daily readings at critical sections. A number of readings should be taken for each load increment to observe the effects of consolidation with time.

Failures

The prime purpose of controlling the rate of embankment construction is to prevent a slide failure and shearing of sand drains or the formation of excessive mud waves.

Failures can be minimized by observing field conditions and the location and formation of mud waves and fine cracks in the embankment. It is then necessary to decrease the rate of construction and increase the rate of

drainage in the critical areas. In order to meet the time schedule, counterberms, flattening of side slopes, wellpoint pumping, or use of lightweight aggregate may be required.

Failures occur when the shear strength of the sub-soil is exceeded. In stratified soils failure may result from pore pressures being transmitted horizontally, reducing effective soil pressures at the embankment toe. It is necessary to take into account the fact that a large portion of the potential slide surface extends beyond the toe of slope, where the benefits of sand drain consolidation and increase in shear strength are minor. From a stability viewpoint alone, it is desirable to drive sand drains beyond the embankment limits, particularly in deep deposits of compressible soil.

Mud waves, located at the toe of slope, may sometimes result from horizontal shear and displacement of the meadow mat, or upper soft soil deposits. The horizontal shear force is caused by the "spreading effect" of the placed embankment.

Hair cracks in the embankment and mud waves are a warning sign of impending failure. Soils stabilized by sand drains are usually of such a nature as to yield plastically before complete failure occurs. A mud wave therefore, is actually a natural counterberm, developed to resist further movement. This natural effect should be assisted by immediately placing material on top of the mud wave to increase even more the counterberm resistance. It is, of course, difficult to prevent formation of all mud waves since soil conditions vary throughout the length of the job and observation points are spaced some distance apart. Daily visual observations are therefore absolutely essential to prevent serious displacements.

In the stiffer more elastic soils it is necessary to build the embankment at a conservative rate as failure may occur suddenly without any prior indications of plastic movement. In the more plastic soils, however, there is some yielding and this serves as a guide to the safe permissible rate of embankment construction. It is also an indication that the embankment is being constructed at maximum rate and the benefits of sand drain stabilization are fully exploited.

Specifications

The specifications and payment provisions for a sand drain job must, in some respects, be rather flexible to take into account the fact that all operations are determined by the soil behavior in the field. Exact quantities are indeterminate and the contractor's operations are often delayed by change orders. In addition, it will be found that many of the measuring devices become damaged or inoperable during the course of the job and have to be repaired or replaced. All possible situations should be visualized and covered by the payment and specification provisions.

Records

In conjunction with field supervision, many records have to be maintained. Besides the usual compaction and soil test records necessary for any road construction work, there are records which are peculiar to a sand drain installation alone. For example, installation records are required of all settlement plates, side stakes and pore pressure measuring devices. Sand drain driving records of length and blows per last foot are required for purposes of payment and future review of driving conditions. It is also necessary to obtain representative driving records for each foot of penetration of some of the individual drains. In addition, the periodic readings of fill height, settlement, gage pressures and side stake movement have to be recorded. Finally,

all of these records must be maintained in graphical form for control of field work and to assist in determining when an additional load increment may be safely added. To facilitate the field work, standard forms should be prepared before the work actually commences.

CONCLUSION

In conclusion, it is apparent that there are many practical details of sand drain design, installation, and field control which must be considered to insure successful completion of the work. As the more desirable sites are developed, the soils engineer will find it necessary to work in areas of poor soil conditions where land is still relatively inexpensive. Although new developments in the future may replace sand drains, they are at the present time an economical means of stabilizing thick deposits of unstable and compressible soil.

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The Author's brother, Henry A. Olko, made all of the drawings.

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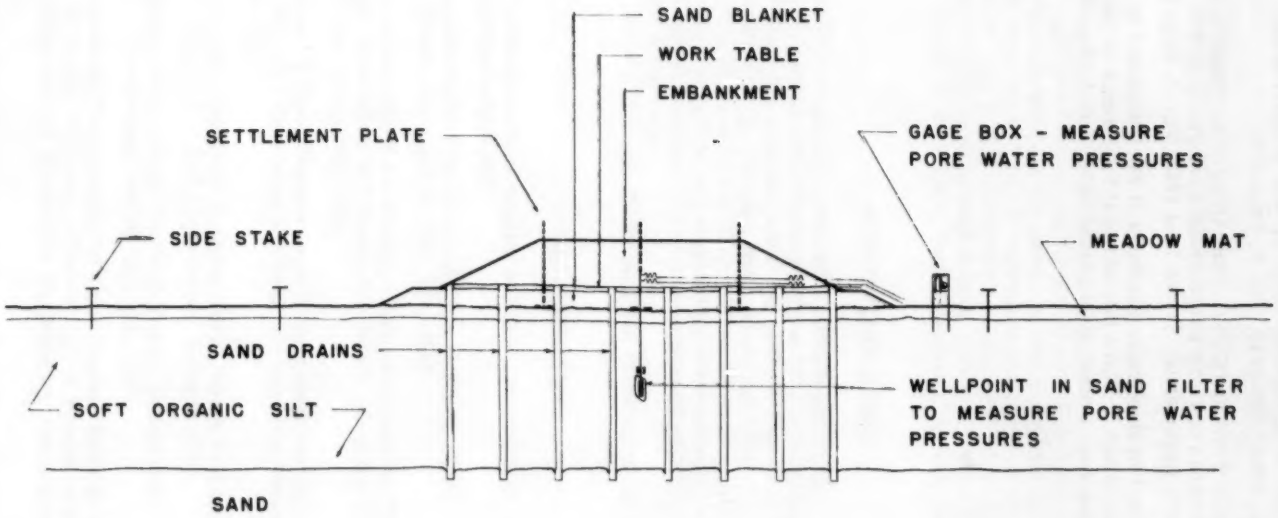
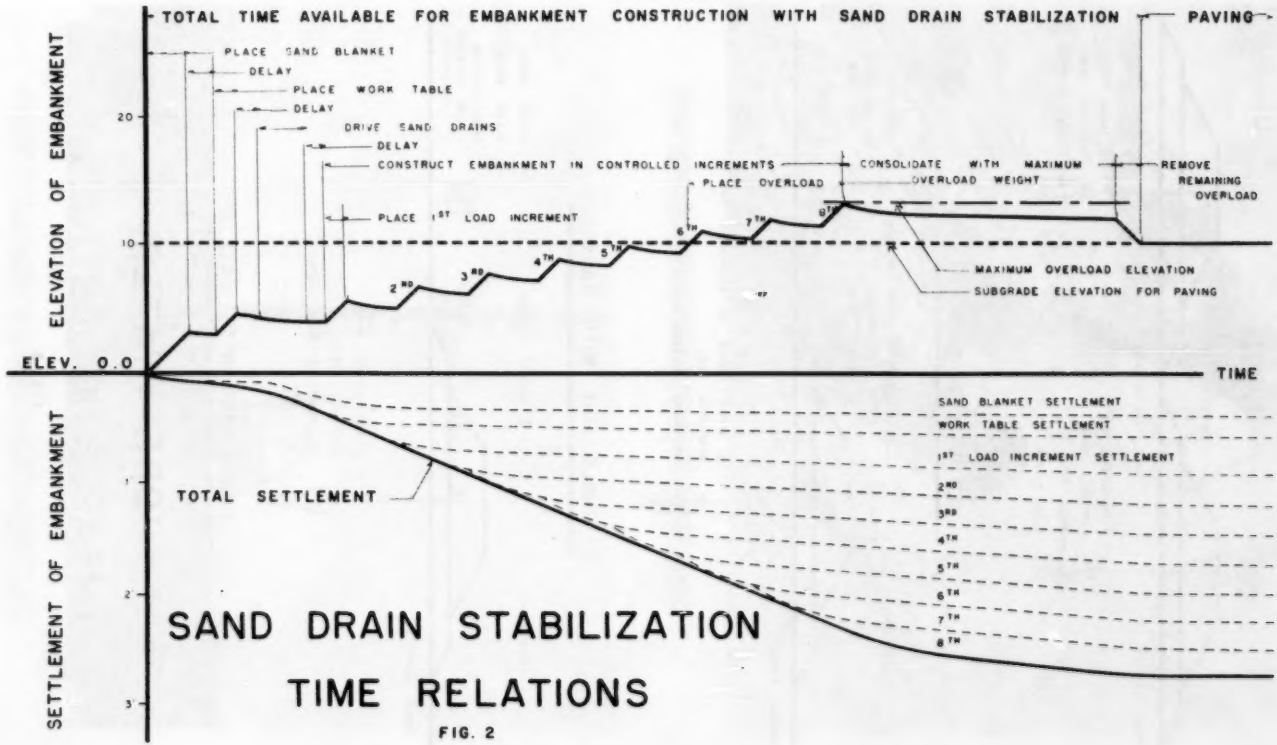


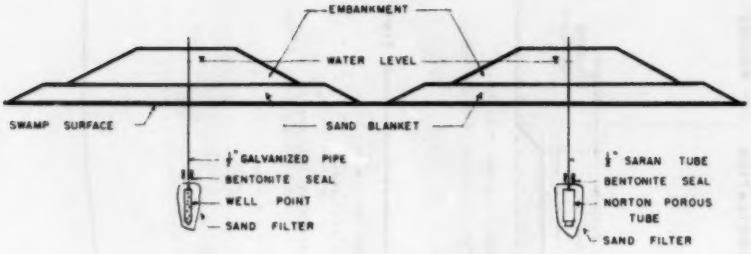
SAND DRAIN STABILIZATION
TYPICAL SECTION

FIG. 1

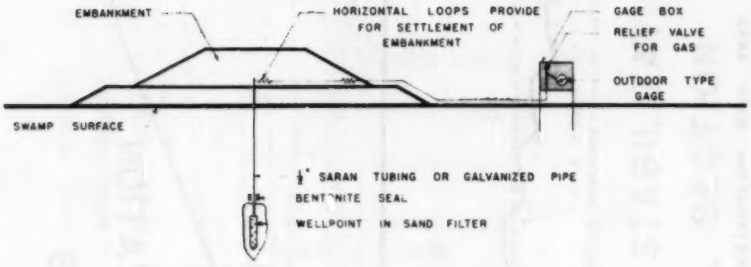


SAND DRAIN STABILIZATION TIME RELATIONS

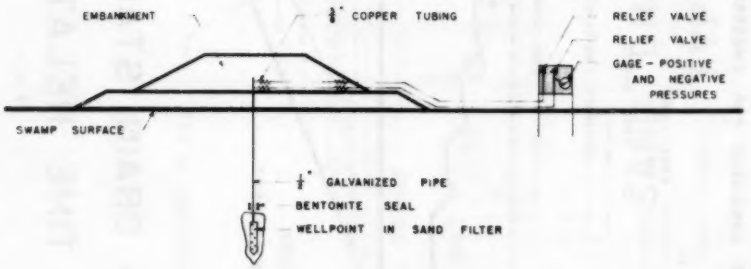
FIG. 2



VERTICAL STANDPIPE



SINGLE LINE WITH GAGE



DOUBLE LINE WITH GAGE

PORE WATER MEASURING DEVICES

FIG. 3

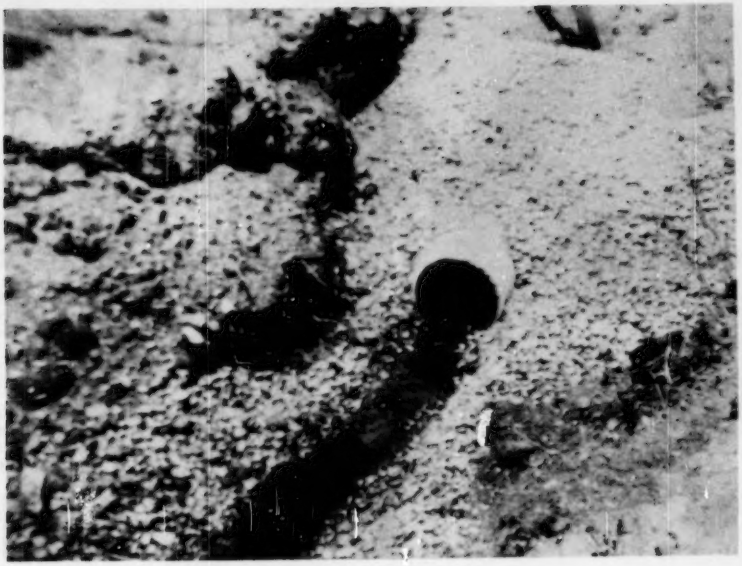


Fig. 4 - Gravel Drain with Concrete Porous Pipe.

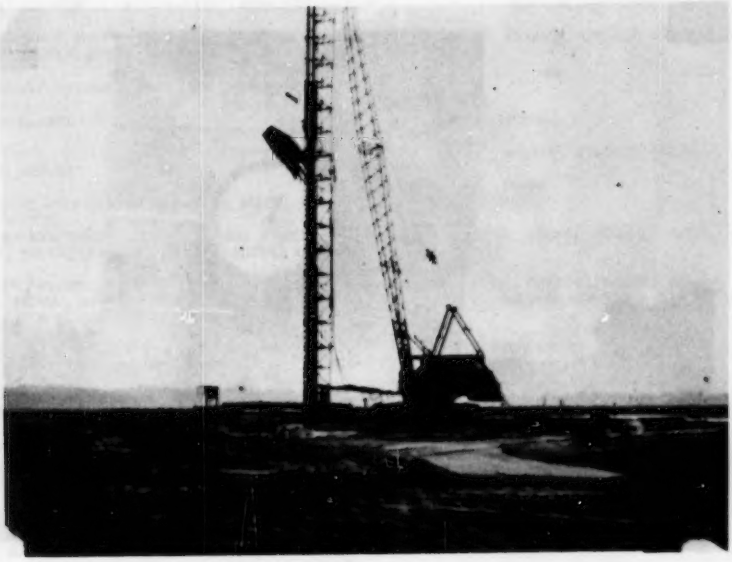


Fig. 5 - Sand Drain Driving Rig With "Skip" Hoist.

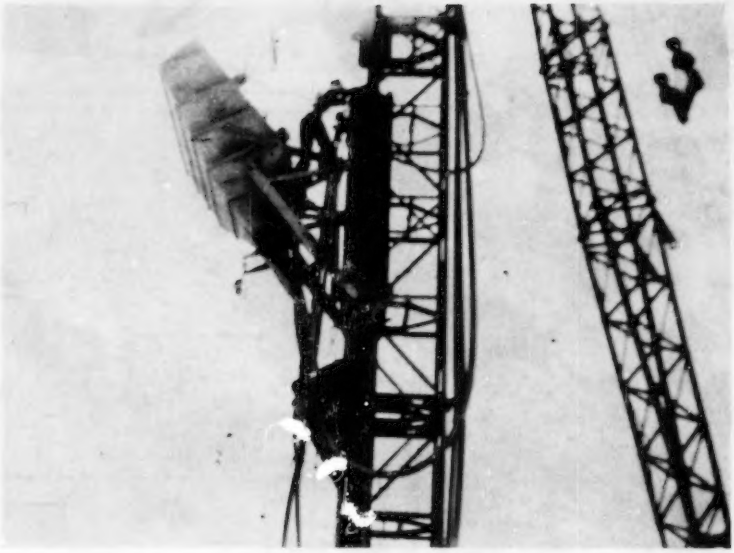


Fig. 6 - Pouring Sand From "Skip" Into Mandrel.

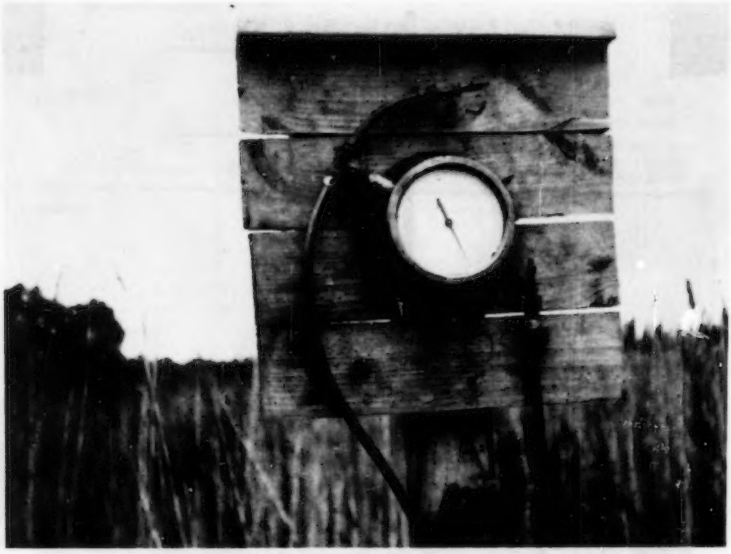


Fig. 7 - Details of Pore Pressure Gage - Double Line System.

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a. Presented at the New York (N.Y.) Convention of the Society in October, 1953.
c. Discussion of several papers, grouped by Divisions.
d. Presented at the Atlanta (Ga.) Convention of the Society in February, 1954.
e. Presented at the Atlantic City (N.J.) Convention in June, 1954.

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